UNITED STATES FEDERAL BUILDING
517 GOLD AVENUE SW
ALBUQUERQUE, NEW MEXICO
BUILDING NO.: NM0024ZZ
PROJECT NO.: ZTX00210

PRELIMINARY STRUCTURAL INVESTIGATION AND ANALYSIS

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1. SCOPE

This report provides a preliminary investigation and analysis of the flat plate floor slabs at the Albuquerque, New Mexico Federal Building performed during the months of December 1992, and January 1993.

The scope of this preliminary investigation and analysis is as follows:

1. Review contract drawings and specifications for the building provided by GSA as well as testing results from recent testing programs.

2. Conduct a site visit to the building to observe the deflections in the floor slabs and gather data on the actual deflections and conditions in the floor slabs.

3. Perform a preliminary structural analysis of the floor structures based on information contained in the contract drawings and specifications for the building, along with test data, in an effort to determine possible causes of the floor deflections and the actual live load carrying capacity of the floor structures.

4. Prepare a report on the findings of the site visit, preliminary findings of the analysis of the floor structures, preliminary conclusions regarding the overall strength and serviceability of the structure, and recommendations for more detailed investigation and study of the structure.

2. BUILDING DESCRIPTION

The Albuquerque Federal Building is an 8 story reinforced concrete structure constructed in approximately 1958. The floor structures consist of 8 inch thick two-way flat slab concrete floor systems with drop heads and shear head reinforcing at the columns. The drop heads are 8'-4" square and 12" total thickness. The drawings indicate that control joints have been placed crosswise in the floor slabs. The control joints are centered between columns and occur at most, but not all bays. The control joints consist of 1/8" thick by 2 3/4" deep continuous steel plates which have been placed in the top of the slabs with the top of the plate being flush with the top surface of the slabs. The drawings indicate that the steel plate is to be notched around any reinforcing steel in the floor slabs to maintain continuity in the reinforcing. A 3/8" deep formed groove is shown on the drawings at the bottom surface of the slabs directly below the steel plate. Underfloor electrical ducts and junction boxes have also been placed in the floor slabs in a number of locations throughout each of the floors. The locations of these underfloor ducts are shown on the electrical drawings. The drawings indicate the top of the underfloor duct is 2 1/2" below finish floor, and is 2 3/8" deep. We have not been able to find anything on the drawings that indicates the width of the underfloor duct. Reinforced concrete columns are spaced at 25'-0" on center in each of the major axes of the slab. The size of the reinforced concrete columns vary from approximately 30" square at the basement level to approximately 14" square at level 8.

The building is rectangular in shape, and is 300 feet long by 102 feet wide. Reinforced concrete bearing walls at the stairs and the elevator core support portions of the floor slabs and also function as shear walls to resist lateral forces. The reinforced concrete columns and walls are supported by a reinforced concrete mat footing which varies in thickness from 1'-10" to 3'-6" thick. The mat footing is thickened to 3'-6" thick below columns and walls. At other areas the mat footing is 1'-10" thick.

Test data received from GSA indicate that rebar at the level 6 floor slabs is Grade 60. The drawings and specifications indicate that concrete in the floor slabs should have a minimum 28 day strength of 3,000 psi with 3,750 psi concrete in the columns. The average strength of three core samples taken from the level 6 floor slab is approximately 2,300 psi which is significantly less than that specified and used for design of the floor slabs. Results of Windsor Probe tests appear to be consistent with the core tests, with an average concrete strength of approximately 2,500 psi.

3. EVALUATION

The floor structures of the building have been analyzed using information contained on the original contract drawings and specifications along with the results of testing conducted on the building under previous studies. The concrete two way flat slabs at the floors were analyzed using the "ADOSS" computer program published by the American Concrete Institute (ACI). This program analyzes and designs two way concrete slab structures using the equivalent frame method and requirements of ACI 318-89. The slabs have been analyzed for the following loading conditions:

Loading Pattern No. 1: All spans loaded with 100% live load.

Loading Pattern No. 2: Two adjacent spans loaded with 75% live load.

Loading Pattern No. 3: Alternate odd spans loaded with 75% live load.

Loading Pattern No. 4: Alternate even spans loaded with 75% live load.

The 25% reduction in live load for pattern loading conditions has been taken in accordance with the provisions of ACI 318-89.

For the purposes of this study, a concrete compressive strength of 2,400 psi has been used in the analysis. This is a general average of the concrete compressive strengths from the core tests and corrected Windsor Probe tests of previous testing programs. The actual concrete strength may be lower or higher than this average compressive strength depending on the location in the slab. Reinforcing steel has been assumed to have a yield stress of 60,000 psi according to previous tests.

The floor slabs have shear head reinforcing consisting of a ring of wire reinforcing at the top and bottom of the slab with inclined and inverted V-shaped #3 rebar at the four sides of the columns, welded around the perimeter of the wire rings. This type of shear head reinforcing is sometimes known as "lampshade" reinforcing and has fallen into disfavor under more recent codes because its ability to develop the necessary stresses and properly anchor the inclined reinforcing on each side of the shear plane in the slab has been seriously questioned. Current codes require that shearhead reinforcing using rebar consist of closed ties with longitudinal bars at each corner of the closed ties.

The shear head reinforcing does not vary in width and is indicated as 2'-2" wide at the top of the slab at all columns. Many of the columns at the upper levels of the building are quite small, and when the critical shear section is taken at a 45 degree angle off of these smaller columns, the shear plane falls very close to the tops of the V-shaped bars. The length of embedment in the slab above the shear plane is very short (approximately 3"), and thus will not provide enough embedment and anchorage to develop the required tensile stresses in the bars across the shear plane. At the best case condition an embedment of only 7 1/2" on each side of the shear plane is provided. Because of this lack of effective development, and the doubt that the shear head reinforcing can function effectively, punching shear in the slabs has been analyzed without regarding the shear head reinforcing.

4. PRELIMINARY FINDINGS

Preliminary findings of field investigation and subsequent analysis based on the evaluation procedures outlined above are as follows:

1. Actual floor deflections measure during the site visit were significantly higher than allowed by the provisions of ACI 318-89. Measured deflections of the slabs on column lines at mid-span varied from 1" to 1 3/4" with respect to the columns. At the large conference room at level 4, the deflection of the slab at mid-span of the middle strip measured 2 3/16". Deflection measurements were also taken using a 4'-0" long level. The amount of floor deflection in the 4'-0" length of the level was measured as shown on the photographs. These deflections varied in different areas of the building from 7/8" to 1 5/8" in 4'-0". Floor deflections have caused cracks in masonry partition walls as well as gaps at the bottom of the walls where the slab has deflected away from the walls as shown on the photographs. Floor deflections have also caused problems with doors in the building as well as requiring furniture to be shimmed as shown in the photographs.

2. From analysis of four of the more typical span conditions at the existing floor slabs, it appears that the flexural slab reinforcing specified on the drawings is sufficient to support the specified 80 psf design live load. A superimposed dead load of 15 psf has been used in this analysis to account for ceilings, mechanical equipment, and other miscellaneous loads. The cut-off points of the flexural reinforcing bars appear to be in accordance with the current ACI code.

3. During the site visit to the building, cracks were observed in the bottom of the slab at approximately 12 to 18 inches on center as shown on Photograph No. 3. These cracks were located near the middle of the slab span and were quite small. The cracks shown in the photograph have been highlighted with a marker so they will

show better on the photograph. Figure 1 shows the crack pattern in the bottom of a test slab at ultimate load. The observed crack pattern was very similar to the crack pattern on Figure 1 for the portion of the slab we were able to observe. It does not appear at this time that the slab has been loaded to ultimate load, but it does appear that the slab has been loaded to the point where it has reached a cracked section which would increase the deflections in the floor slabs.

4. From preliminary analysis of the floor slabs, flexural reinforcing appears to be sufficient under pattern or skip loading using the specified 80 psf live load with the allowable 25 % live load reduction under ACI 318-89 provisions.

5. The 8" thickness of the concrete floor slabs at exterior panels is less than the minimum allowed by present ACI 318-89 requirements A minimum slab thickness of approximately 8.6" should be provided at the exterior panels under present requirements to control deflections. The 8" thickness at interior panels will meet the minimum requirements of ACI 318-89.

6. Based on the findings of preliminary analyses of the floor slabs, and library research into deflections of two way flat slabs, it appears at this time that the excessive deflections in the slab are primarily due to long-term deflections in the floor slabs. Long-term deflections are the sum total of deflections due to elastic deformations, creep, and shrinkage in the floor slabs. Various references have indicated that long-term deflections in two way flat slabs are on the order of three to eight times those measured on initial loading. A good discussion of long-term deflections has been located in the April 1976 ACI Journal, titled "Prediction of Long-Term Deflections of Flat Plates and Slabs" by B. V. Rangan.

   Preliminary analysis of the floor slabs at the columns strips has indicated initial slab deflections at the exterior bays of approximately

1/4" under dead load conditions and approximately 1/2" under total load conditions. At the columns strips at interior bays, the calculated initial slab deflections are approximately 1/8" under dead load conditions and approximately 3/8" under total load conditions. Research has indicated that the differences between deflections of exterior and interior bays are larger only in the uncracked condition, and that on the slab becoming a cracked section, these differences start to diminish significantly. If the calculated initial deflections at the exterior bays are applied to interior bays also, and the deflections are then increased from three to eight times to predict long-term deflections, the approximate long-term deflections would be in the area of the deflections measured during the site investigation.

It is very probable that the creep and shrinkage components of the long-term deflections will be quite high due to the fairly low strength of concrete in the floor slabs. As mentioned earlier, previous core tests have indicated an average concrete strength of approximately 2,300 psi in the floor slabs. The specified strength for concrete in the floor slabs was 3,000 psi in the contract specifications. The lower concrete strength will increase creep in the slabs. It is also possible that the lower concrete strength was caused by additional water being added to the concrete mix. If this was the case, the added water will increase shrinkage in the slab. Increased creep and shrinkage in the concrete will cause increased long term deflections with a minor increase in initial elastic deflections due to a lower modulus of elasticity for the concrete. It is also very possible that the electrical duct system embedded inside the floor slabs is contributing to increased long term deflections due to the decrease in the slab section at the ducts.

7. Preliminary analysis of the slabs has indicated that the punching shear capacity at the supporting columns is not sufficient to support the specified 80 psf live load at levels 5, 6, 7, and 8. This is primarily due to the smaller column size and the associated

decreased area of the critical shear section at these levels. As discussed earlier, the effects of shear head reinforcing in the slabs has been neglected due to serious questions regarding the effective development of the shear head reinforcing on each side of the shear plane.

Punching shear does not appear to be a problem in the floor slabs at the exterior columns due to the presence of spandrel beams around the perimeter of the building, or at the floor slabs around the perimeter of the drop heads. At many of the interior columns, however, punching shear capacity is not sufficient under an 80 psf live load. This is especially true at the first column in from the exterior columns. Table 1 summarizes preliminary allowable live loads on the exterior and interior bays at levels 1 through 8 due to limitations of punching shear capacity in the floor slabs. The reduced concrete strength in the floor slabs has caused a corresponding reduction in punching shear capacity of approximately 10% to 15% depending actual strength of the concrete when compared to the specified 3,000 psi concrete strength.

8. Historical information on the design and testing of two way flat slabs has indicated that the slabs will take almost any load, as long as the load can be effectively delivered to the supporting columns. Even if the flexural reinforcing reaches ultimate load, catenary action in the flat slab will support the load if the load can be delivered to the supporting columns. Proper punching shear capacity around the columns and drop heads is critical to delivering the slab load to the supporting columns. The live load capacity at levels 5 through 8 is limited by the inadequate shear capacity around the columns and the inability to effectively transfer the full 80 psf design live load to the columns without overstress or possible failure.

5. RECOMMENDATIONS

Based on results of our site visit, analysis, and preliminary findings, the following recommendations are made:

1. We recommend that further investigation be performed to view the top surface of the floor slabs around the columns and view the bottom surface of one of the floor slabs for the area of a full bay. The purpose of this investigation would be to view the extent and configuration of any cracking that may be present at these areas. Figures 1, 2, and 3 are taken from test results published in the September 1963 issue of the Journal of the American Concrete Institute. Figure 1 shows the crack pattern in the bottom of the test slab at ultimate load. This crack pattern appears to be very similar to the crack pattern found during our limited observations during the site visit. Figures 2 and 3 show the crack patterns in the top surface of the test slab at design and ultimate loads. We were not able to observe the top surface of the slab during our site visit.

   We would recommend that floor coverings or other finishes be removed on the top surface of the slab to allow us to view the extent of cracking at the top surface. The top surface of the slab should be exposed at a minimum of three columns in the building. One of these columns could be inside the large conference room. The other could be at the columns adjacent to the main hallways. These columns are the first interior columns from the edge of the slag and theoretically have the largest negative bending moment. The floor covering in the hallways could be removed to allow us to view the slab on one side of the column with disrupting office or other areas. The ceiling, and other obstacles where possible, could be removed from one bay of the large conference room to allow us to view the full area of the bottom of the slab at this area.

   All materials and adhesive should be removed to expose the full concrete surface. The locations where floor coverings are removed are somewhat flexible and can be located to cause the least disruptions possible to the building and its occupants. These locations should, however, be at different floors so a more general view of the building conditions can be made. The information gathered through this process is vital to our assessment of the building.

2, In conjunction with the viewing of the slab surfaces above, an effort should be made at a few locations in the building to verify that the actual flexural reinforcing in the slab matches that shown on the contract drawings. This verification can be done by using a magnetic rebar locating device. The rebar locator will provide a fairly accurate indication of the size and location of rebar in the slab. The actual rebar can then be compared to the drawings to get an indication of the adequacy of the flexural capacity of the slab. This verification should be performed at both the top and bottom surfaces of the slab.

3. While preliminary analysis indicates that the range of theoretical long-term deflections in the floor slabs are within the range of those calculated in the analysis of the slabs and measured during our site visit, further research and analysis of theoretical long-term deflections should be performed. Actual calculation of theoretical long-term deflections should be performed using the available materials information after further research on effects of creep and shrinkage parameters on long-term deflections is completed. It will probably not be possible to determine all the characteristics of the concrete in the floor slabs which would effect creep and shrinkage, but a general calculation of long-term deflections should be possible. Some assumptions on actual concrete characteristics may need to be made in the analysis. Variations in concrete characteristics will also cause variations in long-term deflections, so an exact calculation will not be possible. Further study on the effects of the underfloor electrical duct system on the long term deflections in the floor slabs should also be completed.

4. More detailed analysis of the floor slabs at all levels of the building should be performed to expand the preliminary analysis and provide more detailed information on the live load capacities of the floor slabs at various areas of each level.

5. Testing performed under previous studies was performed at level 6 only. We recommend that further testing be performed at other levels of the building to see if the strength of concrete in the floor slabs is consistent with that at level 6. Since punching shear is the controlling factor in the live load capacity of the floor slabs based on our preliminary analysis, an increase in concrete strength above that of the previous tests at level 6 would provide increased shear capacity and live load capacity. This testing program should be done by testing concrete core samples if possible. As a minimum the testing program should consist of at least two cores from each floor level. Additional tests by non-destructive means can be used in addition to the core tests to provide test results over a larger area of each floor level.

TABLE 1

U.S. FEDERAL BUILDING
ALBUQUERQUE, NEW MEXICO

ALLOWABLE FLOOR LIVE LOAD
DUE TO PUNCHING SHEAR LIMITATIONS

LEVEL	EXTERIOR BAYS (PSF)	INTERIOR BAYS (PSF)
8	30	30
7	45	70
6	60	70
5	70	70
1 to 4	80	80

15 psf superimposed dead load has been used in the analysis of the slab.

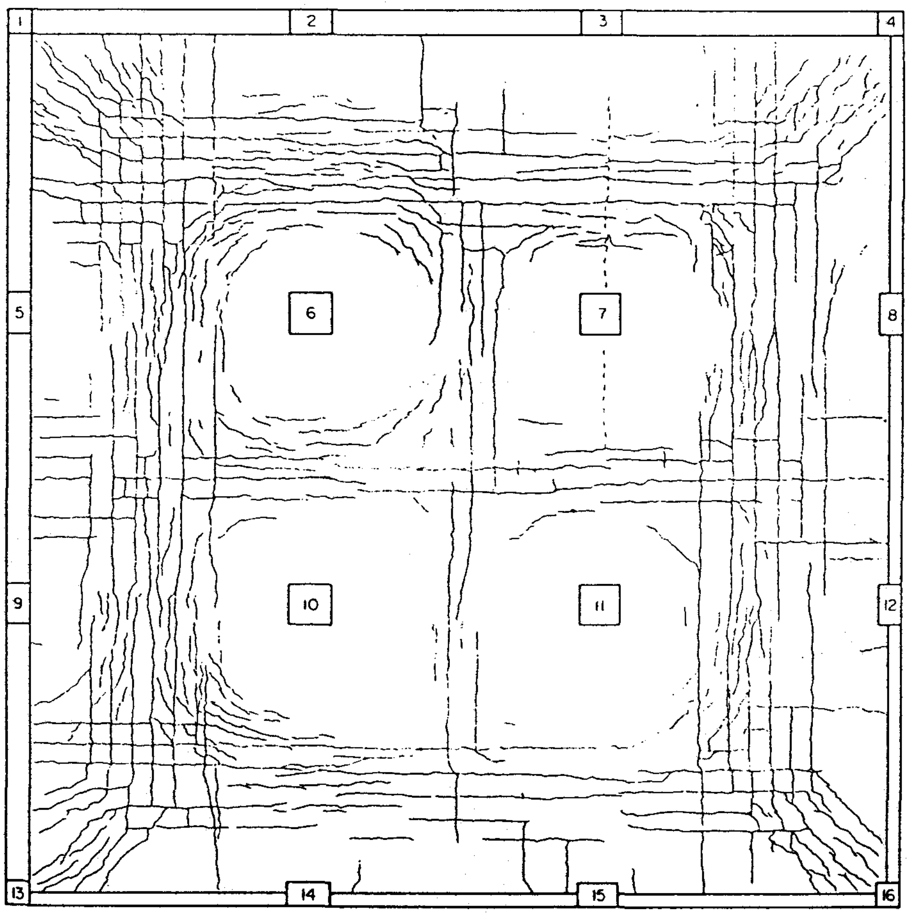

Fig. 1 —Bottom surface crack pattern at ultimate load

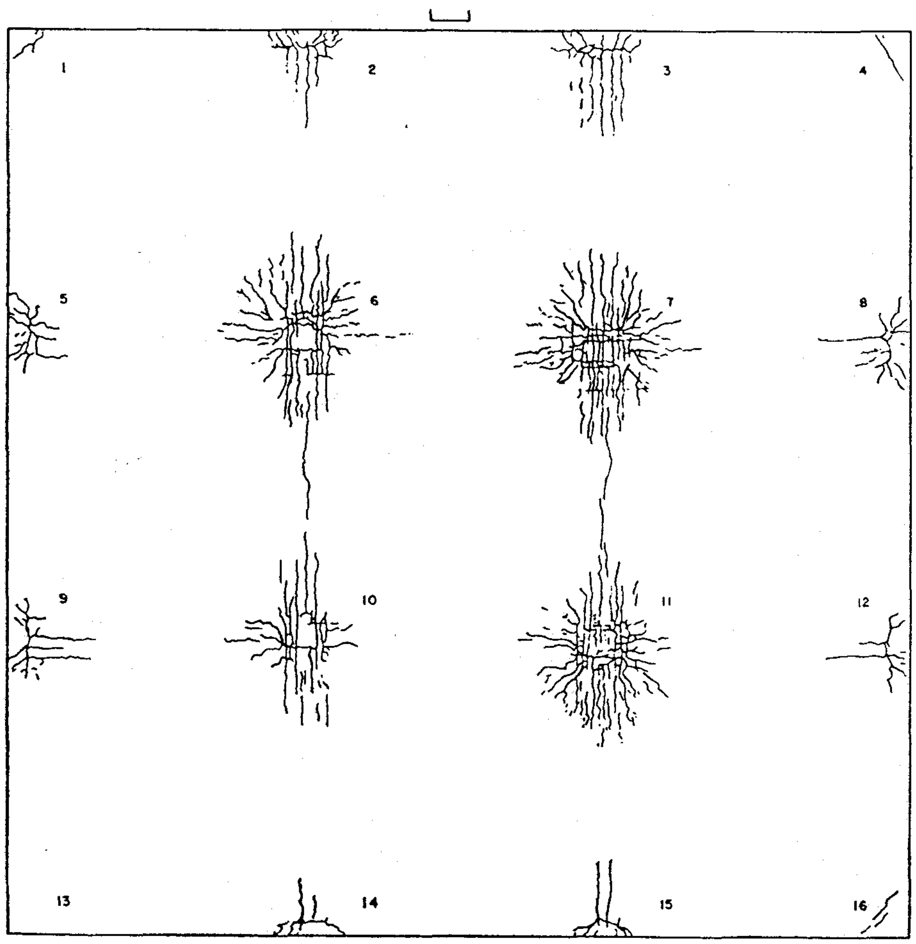

Fig. 2 —Top surface crack pattern at design load

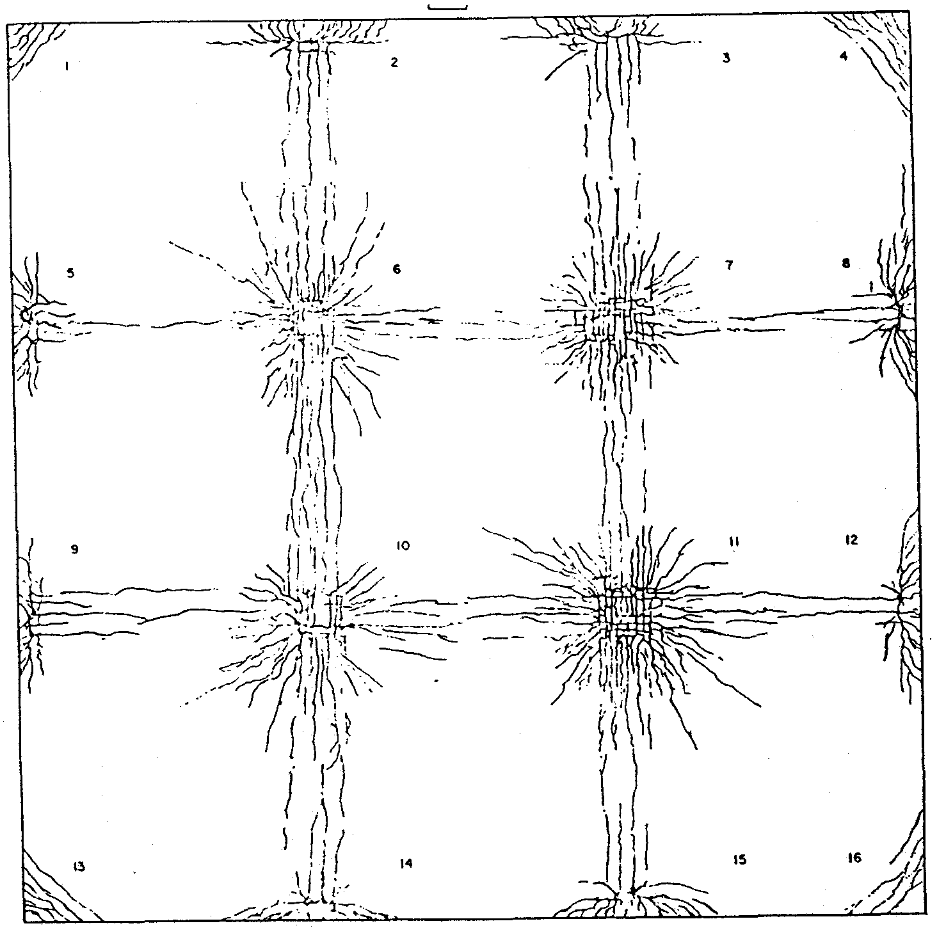

Fig. 3 —Top surface crack pattern at ultimate load